COLCRETE

COMMERCIAL - IN - CONFIDENCE

# WAVE ENERGY STUDY

## NEL OSCILLATING WATER COLUMN (WAVE PISTON)

## BREAKWATER DEVICE

Report by Colcrete Limited, Rochester on the use of rock anchors for providing long term stability of the breakwater device.

JANUARY, 1980


Executive Summary

This report outlines the requirement for permanent rock anchors to stabilise Breakwater Devices, discusses and confirms Roxburgh & Partners' choice of 3610kN anchors and provides preliminary design calculations and estimated costs in support. The permanence and service life of the anchors are examined and found satisfactory. The report goes on to describe working methods, plant and labour requirements and programme, it comments on the use of anchors in weaker materials. Details of further information required for optimum design and recommendations with costs are given for a test anchor programme.



Roxburgh & Partners,
Mirren House,
6 Maxwell Street,
Paisley.

National Engineering Laboratory,
East Kilbride,
Glasgow.

## 1. Introduction

This Report follows on from Roxburgh and Partners Preliminary Study Report No. PR7:Y5 DEY2 dated May 1979 and examines in detail the feasibility and economics of using Rock Anchors to provide long term stability to NEL Oscillating Water Column Breakwater Devices.

The purpose of these devices is to convert wave energy into electrical power suitable for supply to the National Grid. In principle an "air over water" wave piston is created within a concrete structure. The reciprocating air flow thus produced is rectified to provide a unidirectional flow which passes through an air turbine and drives an electric alternator. Each structure will contain four wave piston units, will measure approximately 77 x 47 x 32m high and is designed to be fixed to the sea bed in shallow water inshore sites with a 15 - 20m depth of water. The concrete structures will be substantially constructed in oil platform dry docks, and floated to their final location and installed side-by-side to form continuous breakwaters. Float out and installation on site will be undertaken during suitable "weather windows" but, although ballasted in position prior to fixing of machinery modules and completion the structures will be inherently unstable in stormy conditions.*

To ensure the operational safety of each four cell structure it is calculated that a total additional design working force of approximately 680,000 kN is required.

The most practical and economic method of providing this additional force on location is by means of permanent rock anchors. Whilst successfully used since the 1930's permanent rock anchors have, over the last fifteen years, been developed to meet the exacting requirements of the civil engineering industry in terms of performance and long term reliability.[1,2,3,4,5]
They can be installed and tensioned comparatively quickly, thereby minimising the period during which individual structures are exposed to risk of damage or displacement by storms. Because of their small size in relation to

working force the installation equipment is also correspondingly small and can be easily accommodated on the structure itself. However, since other critical activities will be proceeding concurrent with anchor installation there may be some advantage in locating most of the ancilliary equipment on a work barge or similar.

Suitable sites for the Breakwater Devices are found around the Scottish coast and on the South West coast of England. They must have the required "exposure" to suitable wave conditions, a 15 to 20m depth of water and a sound hard rock sea bed.

* Footnote:

We understand that the Consultants are now proposing to provide stability during the critical stages of installation by means of a catamaran type emplacement barge.

## 2. Selection and Sizing

By reference to Roxburgh and Partners' Drawing No. 3RD/1 it is apparent that the total anchoring force required for a four cell structure is 678,680kN. In theory this force could be provided by the 68 No. 10,000kN or 1,000 No. 680kN anchors or any intermediate capacity and number that would produce the same total force.

The selection of anchor capacity must be made having regard for:-

a) The current availability of installation and tensioning equipment. As 10,000kN anchors are outside the range of anchors normally constructed, special equipment would have to be developed and this would not be available for the proposed test anchor programme.

b) The economic design of the structure. A complete reappraisal of the design would be necessary if high capacity anchors at wide centres were to be used.

c) The available installation time. If a large number of low capacity anchors were used because available working space would limit the number of drill rigs,the installation time and hence the period during which the structure was potentially at risk would increase by a factor in excess of three. This would be unacceptable.

Therefore the decision to adopt anchor working loads of 3610 kN is soundly based, for such anchors are within the easy practical grasp of current British contracting skills, and a more widespread distribution of anchors would probably not serve to distribute the foundation pressure more evenly in view of the rigidity of the foundation. Further, the capacity chosen, bearing in mind test loading to 1.5 times working load, should not necessitate major changes in the nature of the structure itself to accommodate high punching loads other than helical reinforcement associated with the top anchorage stress distribution.

For anchors of the capacity and length required, steel tendon of multi-strands represents the optimum solution in terms of cost and handling in these conditions.

With anchors at 1.25m centres, as indicated on Drawing 3RD/1, and 42m deep it would be normal practice to deviate alternate anchors in opposite directions to avoid the possibility of anchor holes intersecting whilst still within the recognised 1 in 50 drilling tolerance. In this instance, however, because the upper 23m of each hole will be sleeved through the structure and only 19.0m actually drilled in rock the anchors can all be drilled in the same plane, provided that the sleeves are accurately placed in the structure. Where anchors are installed in rock having horizontal joint or bedding planes it is advisable to stagger adjacent fixed anchor zones vertically by 2 - 4 m to reduce stress concentration and prevent laminar failure. Bearing in mind the fundamental importance of the rock anchor system to the overall success of the project, thorough engineering, meticulous pre-planning and carefully executed and controlled site work are considered essential. The use of medium capacity 3610 kN tendons employing design techniques, construction methods and materials currently available should not evoke special problems. It is thought prudent, at this stage, to simplify the "engineering" so as not to add to the undoubted practical and logistic problems that will result from working in the extremely exposed locations where the devices will be situated.

Our preliminary design for the anchors is given in Appendix II and, taking account of the probable location of the fullscale sea trial assumes the rock to be Lewisian Gneiss. To limit the working stress in the tendons to 50% f p u they would consist of 29 No. 15.4mm diameter Supa strands. Based upon drilling 215mm diameter anchor holes the fixed anchor length is calculated to be 3.8m but without any information about the rock this has been arbitrarily increased to 6.0m. To cater for the overall stability of the system the anchors should penetrate up to 15m into the rock. With the exception of the overall stability calculation, which equates the submerged weight of rock mobilised with the anchor working load but takes no account of the shear strength of the rock,[12] the lowest factor of safety in the system is 2.0 and applies to the tendon steel, the ultimate strength of which is accurately known. A factor of safety of 3.0 is applied to the rock/grout bond but it should be noted that the upper limit of compressive strength assumed for the rock is a conservative 42N/mm$^2$, with an implied working bond strength of 1.4 N/mm$^2$.

## 3. Anchor Hole Drilling

With the present design of a four cell structure there are a total of 188 No. 3610 kN anchors of which 152No. are vertical and inclined in the walls and 36No. are inclined in the base of the water columns. With these anchors installed and stressed the overall factor of safety against sliding is 1.5. In order to secure the structure to the sea bed with a factor of safety of 1.0 it has been calculated that 122No. vertical anchors will be required. The prime objective in the choice of installation techniques and materials therefore must be to install these anchors with maximum speed and efficiency. The two critical factors that control the period before the anchors are tensioned and in service are drilling of the holes and curing of the grout.

There is no doubt that the quickest and most efficient method of drilling 215mm diameter holes in strong rock is with down-the-hole hammers. This technique uses a percussive drilling hammer immediately above the drill bit. The hammer is powered by high pressure compressed air led through large diameter drill rods or tubes. The energy losses are minimal compared with "top drive" hammer drills and drilling accuracy and output are much improved. Rotary action is imparted by the drill rig located at the top of the hole, via the drill tubes.

Conventional and commercially available drilling rigs capable of drilling holes of this diameter to depths in excess of 40m are generally track mounted and have either compressed air or diesel hydraulic prime movers. These types of rigs would be used for any test or prototype work. In operational mode they require 3 to 4m working distance on one side of the hole, in which to position the rig, and a 2-3m man access width in front of the rig for handling the hammer and tubes. Looking at the production phase for at least 1500 No. structures, special equipment could be developed to reduce working space requirements and take advantage of available power sources. The high pressure(11-14bar) compressed air requirement for hammer operation and flushing would however, remain the same.

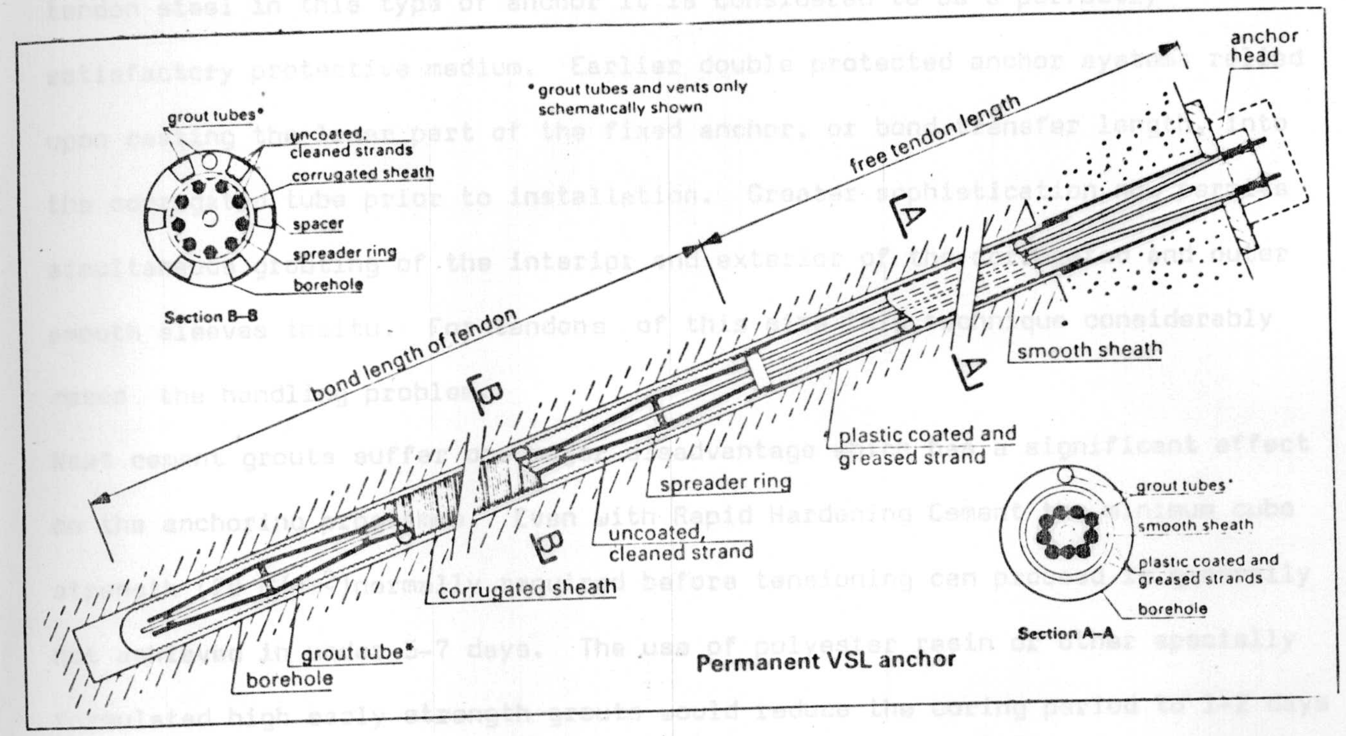
To utilise the high output available from this type of equipment it is essential to avoid the use of temporary casing to ensure the hole is "sealed", reduce water ingress and allow proper flushing of drill cuttings. Although the anchor holes will be sleeved through the structure during construction and although in rock of reasonable quality the holes can be expected to remain stable without casing, the section of the hole between the underside of the concrete structure and the prepared sea bed will provide a leakage path unless special measures are taken.

This seal could be achieved by underbase grouting [9,10] but, with the time required to ensure grout tight seals around the base of the structure on location and, in view of the anticipated grout volume, the time required for grouting, if this was done prior to rather than concurrent with anchor installation there would be an unacceptable extension of the period during which the structure is at risk.

Provided the upper surface of the rock on the sea bed could be prepared to a tolerance of say ± 150mm a practical alternative method of sealing the holes would be to cast purpose fabricated annular ring seals around the anchor holes on the underside of the structure. Using a woven synthetic fabric these seals would be "inflated" with grout once the structure is in position thereby "lining" the hole to the top of the rock. Final grouting with a sand/cement grout through and at the bottom of the sleeves would complete the sealing operation.

## 4. Corrosion Protection

In accordance with the recommendations of National and International Standards for permanent anchors and taking account of the potential aggressivity and long term importance of the anchors, protection against corrosion is considered essential. In essence this means that every part of the tendon and anchor head must be surrounded by at least two media that will provide a positive barrier to the active ions that promote corrosion.




FIGURE

The type of anchor proposed is illustrated in Figure I. With this type of anchor the load transfer or fixed anchor length is encased within a corrugated plastic sheath containing grout.

This corrugated protection would consist of suitable heavy duty grade polythene of appropriate diameter to accommodate the tendon configuration. Specific recommendations can be made when the anchor design and construction have been optimised. The free length, the individual strands are individually greased and sheathed in polythene (1-2mm thick) and then enclosed in a plain polythene outer sheath of diameter approxiamtely equal to that of the corrugated fixed anchor protection, the junction between the two being formed by a suitable heat shrink plastic bonding. Hence the free length has a high degree of protection to match that of the fixed length. The judicious selection of outer polythene tube diameter and borehole diameter to ensure a subsequent grout annulus of at least 10mm thickness provides another stage of corrosion protection to the whole system.

The most suitable type of grout from the dual standpoints of ease of operation and cost is neat cement. In the context of the overall protection given to the tendon steel in this type of anchor it is considered to be a perfectly satisfactory protective medium. Earlier double protected anchor systems relied upon casting the lower part of the fixed anchor, or bond transfer length, into the corrugated tube prior to installation. Greater sophistication now permits simultaneous grouting of the interior and exterior of the corrugated and outer smooth sleeves insitu. For tendons of this size this technique considerably eases the handling problem.

Neat cement grouts suffer one major disadvantage which has a significant effect on the anchoring programme. Even with Rapid Hardening Cement the minimum cube strength (28 N/mm$^2$)normally required before tensioning can proceed is generally not achieved in under 5-7 days. The use of polyester resin or other specially formulated high early strength grouts would reduce the curing period to 1-2 days but would add considerably to the risk of faulty construction, as these materials are difficult to handle and pump in large volumes, and also to the overall anchoring cost. Once again special grouts and equipment could be developed for the production phase of the project, if this saving in time is thought to be critical.

Unless there is a specific reason related to the structures, the prepared foundation or a requirement to remove the units at the end of their service life there does not appear to be any advantage in providing the anchors with a facility for restressing or destressing.

The long term behaviour of production rock anchors is a question of extreme relevance. Littlejohn and Bruce (1976) discuss the problem and have just published results from a study at Devonport.[4,5.] In summary it seems that, provided anchors are installed into good competent rock (and the Lewisien Gneiss, if fresh, certainly falls into this category) the prestress lost due to non-tendon phenomenon is relatively small.

Of course, Supa strand, as recorded, has excellent long term load holding characteristics and these are amply detailed in the relevant brochures as supplied by Bridon Wire.

Even where our present predictive capacity suggests that in service load losses may be very small, tendons are locked off at working load plus 10% as a safeguard. Furthermore, it is now commonly realised that restressing tendons after a certain period of time reduces substantially subsequent prestressing losses in service. This is an option which can be exploited prior to cutting and capping the anchored heads.

The inference is therefore that with the anticipated nature of rock, the design of the tendon to operate at 50% fpu, and a carefully planned and executed stressing programme, the long term behaviour of these anchors should not present a problem.

Anchors of this type are currently designed to have a service life in excess of 50 years and there is no reason to expect the properly fabricated double protected tendons installed without damage should not considerably exceed this period.

## 5. Anchor Installation

Following the drilling of each borehole, it would be simply water tested. If the hole failed a certain criteria e.g. 10 Lugeons[11] it would be grouted and re-drilled after not less than 24 hours. Dense low water cement ratio grout would be used throughout the grouting operations in view of the seawater environment. The grouting process should be repeated until an effectively 'tight' borehole is achieved. It should also be borne in mind that corrosion is generally dependant on the renewal of the supply of active ions to the surface in question. The borehole water proofing procedure further precludes the possibility of continual 'washing' of any corrodable materials by corrosive solutions.

To achieve the necessary degree of quality control the anchor tendons will have been fabricated at an onshore 'factory', where they will be coiled on special 2.5 to 3.0m diameter drums for delivery to the structure. When the hole has been satisfactorily drilled, flushedout and water tested the precoiled tendon will be loaded vertically into a hydraulically operated handling machine located at the top of the structure and lowered carefully into the hole.

To allay any fears that tendons might sustain damage during homing, a few randomly selected tendons can be withdrawn from their boreholes immediately after homing but prior to grouting to permit damage assessment. The presence and type of damage if any dictates what changes in the installation technique are required.

If Rapid Hardening Cement is found acceptable in relation to programme requirements it will be mixed with clean fresh water in a Colcrete Colloidal Mixer. The water cement ratio will be carefully controlled in the 0.40 to 0.45 range and, to improve the rate of gain of strength a water reducing plasticising admixture will be used. The mixed grout will be pumped into the internal and external annuli through tremie tubes incorporated in the tendons. To permit the necessary control of pumping pressures hydraulic Evans pumps would be used.

It is envisaged that the inclined anchors in the bottom of the water column chambers would be drilled, installed and grouted in the same manner, but the tendon handling unit would remain at a high level. Some form of "stuffing box" will also be required to resist the additional hydraulic pressure head during the drilling and pregrouting operations. Once the anchor grout has achieved the specified minimum strength the anchor heads would be fitted and proof testing and stressing would commence using multi-strand jacks. Each anchor would normally be proof tested incrimentally to 1.5 times working load in at least two cycles before locking off at 1.10 times working load. Check-lifting after 24hours and say 2-4 weeks would monitor any initial load loss and enable load adjustment prior to trimming and capping the tendons. For practical reasons this procedure could be relaxed for the anchors inside the water column chambers.

## 6. Programme and Installation Time

With the majority of anchors vertical and sleeved 23m through the structure, and with up to 20m of rock drilling in rock of reasonable quality, we would expect an output of two anchors per 12 hour shift per rig. If extensive pregrouting and redrilling proves necessary additional rigs would be required.

In view of the location of the work we would think it prudent to apply a 20% contingency factor to double shift production which would thus be reduced to 3.2 No. anchors per day per rig.

The eventual decision on how many rigs to employ will be influenced by many factors including, site location and risk, space available on the structure, space and accommodation on support vessel and other concurrent offshore construction actitities on the structure. These factors will probably also vary according to geographical location.

Allowing a one week curing and tensioning period as a constant factor the 122 No. critical anchors could be installed in the following periods dependant upon the number of rigs employed.

2.9 Weeks with 3 No. Drilling rigs.
2.4 " " 4 No. " "
2.1 " " 5 No. " "
1.9 " " 6 No. " "

Total time for installation and initial tensioning, excluding check-lifting, of the 188 No. anchors in a four cell unit will therefore be:-

4 Weeks with 3 No. Drilling rigs.
3.3 " " 4 No. " "
3.0 " " 5 No. " "
2.6 " " 6 No. " "

This assumes that the average output on the 36 No. inclined water column chamber anchors is similar to the longer wall anchors, to take account of the more difficult working conditions.

## 7. Working Space Requirements

The total working space required by the anchor installation and testing equipment is dependant upon programme considerations and the number of drilling/installation units to be used. Taking a two rig unit as an example the following major items of plant would be required.

2 No. Drilling rigs.

1 No. Tendon handling unit.

2 No. Sets stressing equipment.

All these items would have to operate from the structure itself. In addition the following plant would be required, but this could work from a barge or other support vessel.

2 No. 750 cfm 11-13 bar Compressors.

1 No. DD4 Colcrete Mixer

2 No. 9" x 2" x 12" Colcrete Evans Pumps

Adequate spare units would be considered essential to maintain a tight programme together with storage facilities for tendons, cement, fresh water and consumable spares.

Accommodation at sea for labour and supervision would be required. In this connection a five man crew would operate one complete drilling/installation unit per shift with approximately one supervisor or engineer per unit. Thus the total labour requirement for a six rig two shift operation would be approximately 72 men.

Accommodation, messing, fuels, power, lighting, weather protection and working access would normally be provided by the main civil contractor.

## 8. Cost of Proposals

The approximate cost for the supply, installation and testing of permanent anchors in accordance with the above is as follows, based upon September, 1979 prices.

| | | |
|---|---|---|
| a) Provide, mobilise and establish plant and equipment on vessel at shore base. | | |
| | Per visit | 45,000 |
| b) Provide, prepare in onshore factory and coil 3610kN double protected anchor tendons comprising 29 No. 15.4 mm Supa strands. Together with cement for grouting and anchor heads. | | |
| | 188 No.anchors @ £2,000 | 376,000 |
| c) Drill open hole in rock, install 3610 kN rock anchors not exceeding 45m long, mix and inject grout using neat cement, proof test and lock off. | | |
| | 188 No.anchors @ £400 | 75,200 |
| | | £496,200 |

The establishment item (a) is based upon working on a single structure. If continuity of anchoring operations from structure to structure was possible moving and relocation would be charged at an hourly rate basis per rig unit hour or per operation if the time could be quantified prior to the event. Delays due to weather would be on the same basis.

The approximate cost per rig unit per day on a dual shift basis is £1,200.

## 9. Ground Anchors in Other Types of Material

Anchors have for many years been installed in materials other than strong rock 1,2,3,6,7,8 and the current state of the art is covered in Littlejohn's 1979 appraisal.

As all anchors whether in rock or soil derive their ultimate capacity from the insitu strength of the material surrounding the fixed anchorage it follows that, for the same anchor dimensions, there will be a reduction in available capacity in weak rocks, non-cohesive and cohesive soils. In the context of the structures being considered the number of anchors will increase rapidly as the design working load reduces. In certain weak rocks such as the lower quality Grades of Chalk and Marl and in clays where pore water pressure dissipates slowly it is normal to increase the design factor of safety when determining the fixed anchorage length; in order to compensate for creep losses within the ground.

Once again there are practical difficulties in increasing the anchor hole diameter and it would appear that 200mm is the optimum but in granular soils which have to be temporarily cased full depth, this may have to be reduced below 150mm. Owing to the fact that percussive down-the-hole equipment cannot be used in these materials production rates are substantially reduced and one would not expect to exceed more than one anchor per shift per rig. This coupled with the increases in the number of anchors would result in the anchor installation programme being extended considerably.

As an indication the maximum safe capacities that can be expected in various types of soils based on 200mm drilled holes:-

| | |
|---|---|
| Marl, Grade I & II | 1500 kN |
| Chalk, Grade I & II | 1500 kN |
| Coarse Gravel | 1200 kN |
| Gravel & Sand | 900 kN |
| Fine Sand | 600 kN |
| Stiff-hard Clay* | 1000 kN |

*With clays this capacity could only be achieved by undereaming.

R

## 10. Conclusions

The fundamental proposal to use permanent rock anchors to provide the additional force required to prevent the wave piston structures from sliding is entirely feasible. Indeed it is difficult to envisage how, without increasing the mass of the structures significantly the design stability could be achieved quickly on location without the use of rock anchors.

The present design incorporating 188 No. 3610 kN anchors up to 42 m long does not raise practical or technical problems outside current experience.

A smaller number of higher capacity anchors is not considered viable within the time scale proposed as special equipment would have to be developed and put into service without sufficient field experience.

Dependant upon overall programme requirements, available working space and hence the number of drilling rigs employed, the first 122 No. critical anchors in a four cell structure could be installed and tensioned in 2 - 3 weeks with anchor completion in a total of 3 - 4 weeks. These periods could be reduced by 3-4 days if very high early strength grouts were used but there would be a significantly increased risk of technical problems during anchor construction and the probable"loss" of some anchors.

Provided adequate attention is paid to corrosion protection service life of the anchors can be expected to exceed 50 years. After initial load loss which is compensated for by the lack of load used in practice performance during service should be satisfactory.

Although all the anchoring plant could operate from the structure there is a probability of congestion, due to other activities, and it is therefore recommended that all ancilliary plant, storage facilities and accommodation are provided by a support vessel. Whilst anchors could be used for locations where weaker materials exist below the seabed the capacity would be reduced and the number of anchors increased by factors of at least two. Also the period during which the structure was at risk from storm conditions would be more than doubled.

The approximate cost of the rock anchors for one device assuming September, 1979 prices and no continuity to the next structure would be £500,000.

The Appendices to this Report set out a list of geotechnical data required for design purposes, our design calculations, proposals for a preliminary test anchor programme and a list of references.

APPENDIX I

Geological and Geotechnical Data Required for

Detailed Design.

To enable the final design to be carried out for anchors in the Test Area and Wave Piston Device Locations, comprehensive Site Investigations are required since it is of prime importance to have detailed knowledge of the ground.

For structures of this area a minimum of four and preferably six boreholes should be put down to a depth of 25-30m below the seabed.

The geometry of an anchor system and its mode of operation requires, in particular, knowledge of ground conditions local to the grouted fixed anchor zone. Discontinuity frequency and orientation data together with joint continuity and roughness can be vital in determining the size and shape of a rock mass liable to fail in service and therefore are critical in any overall stability analysis. In practice these data and parameters such as Rock Quality Designation can be invaluable when back analysing water test data to determine the need for pregrouting.

Emphasis must be placed on obtaining maximum continuous core recovery, which generally implies core diameters of not less than 75mm. In addition, the use of double or triple tube core barrels is recommended. For weak rocks which are difficult to core, the SPT has been exploited to give a relative measure of insitu quality. Stress/strain characteristics e.g. 'E' values, are important in design since they influence bond distribution, and may dictate the failure mechanism in the fixed anchor. For strong rocks the Goodman Jack is appropriate although if results are difficult to determine, deformability measurements from cores should be seriously considered.

Determination of the groundwater conditions will almost certainly be essential for the overall design of the project as well as the anchor system. In hard rock where low permeability is confirmed it is noteworthy that the environment is is sometimes regarded as virtually non-aggresive on the basis of low groundwater percolation rates.

In rocks, index tests such as Point Load Strength are attractive since they are cheap, easily undertaken and correlate approximately with other parameters such as uniaxial strength, although concurrently it is more common to determine directly the uniaxial compressive strength or occasionally the tensile strength on specially prepared test cylinders. Alternatively, a large shear box may be used to assess the shear strength of intact material or an existing discontinuity. The shear strength of joints may also be estimated by a detailed study of the joint geometry and materials characteristics. All these tests are used by the designer to estimate rock mass stability, and the bond or skin friction in the fixed anchor zone. In regard to bond distribution however, rock deformability is the key parameter, and stress/strain relationships should be obtained from uniaxial compression tests in the laboratory, or preferably from insitu pressuremeter tests.

The susceptibility of rock to weathering can be assessed by the Slake Durability test apparatus, augmented by a microscopic examination of the nature of the minerals. With this information the sensitivity of the rock to flushing water and the possibility of mineral reaction with grout or groundwater can be investigated. In this regard swelling tests are also pertinent. In general there is a need to concentrate more on ground parameters which affect or may be affected by the drilling prcess. For example, quartz content combined with strength are useful figures when assessing drillability.

Sulphate and chloride contents are established as a routine and dictate choice of cement, but the overall corrosion hazard is seldom quantified. As a guide Table I illustrates some aggressivity limits with respect to cement, whilst Table II proposes limits for two key parameters with respect to metals.

Table I : Aggressivity of groundwater with respect to cement

| Groundwater Environment | Remarks on Aggressivity |
|---|---|
| Very Pure Water<br>(CaO < 300mg/litre) | Such waters dissolve the free lime and hydrolyse the silicates and aluminates in the cement. |
| pH < 6.5 | Acid waters attack the lime in the cement, but pH values of 9-12 are passivating. |
| Selenious water ($SO_3$) > 0.5g/litre (stagnant)<br>> 0.2g/litre (flowing) | These sulphates react with the tricalcium aluminate to form salts which disarrange the cement by swelling |
| Magnesium water ($SO_3$) > 0.25g/litre (stagnant)<br>>0.1g/litre (flowing | |

Table II : Aggressivity of soils with respect to metals.

| Ground Environment | | Remarks on Aggressivity |
|---|---|---|
| Resistivity ohm cm | Redox potential (corrected to pH = 7) Normal hydrogen electrode mV | |
| < 700 | < 100 | Very corrosive |
| 700-2000 | 100-200 | Corrosive |
| 2000-5000 | 200-400 | Moderately corrosive |
| > 5000 | > 400<br>> 430 if clay soil | Midly or non-corrosive |

APPENDIX II

Design calculations for 3610KN working load anchor

N.B. Calculations based on Drg. No. 568/PGA 21/1 Rev A

issued with the Preliminary Report PR7.

1. Tendon

Select 15.4 mm dia. supa strand.

Specified characteristic strength fpu = 250 kN.

Select safety factor = 2.0

Therefore, no. of strands per anchor is

$$\frac{3610 \times 2}{250} = 29$$

2. Fixed Anchor Length

(a) Strand to grout interface

Minimum uniaxial compressive strength of grout $40N/mm^2$

at 28 days.

Ultimate bond stress taken as $2.0N/mm^2$.

Select safety factor = 2.5

Then working bond strength = $0.8N/mm^2$.

and minimum length of fixed anchor to transfer

stress from strand to grout is

$$\frac{3610 \times 10^3}{\pi \times 15.4 \times 29 \times 0.8} = 3216\text{mm}$$

c.f. Transmission length for small diameter strand working

at 70% fpu in concrete of strength 35 to 48 $N/mm^2$ which was

found experimentally to be 25 to 31 diameters i.e. 385 to 477mm.

(b) Grout to rock interface

(i) Select borehole diameter such that

$$\frac{\text{Area strand}}{\text{Area borehole}} \leq 15\%$$

R

$\frac{15.4}{2}$

i.e. $$\frac{\pi \times 7.7^2 \times 29}{\pi \times R^2} \leq 0.15$$

or $$R \geqslant \sqrt{\frac{7.7^2 \times 29}{0.15}} = 107.1\text{mm}$$

Therefore use a borehole diameter of 215mm.

(ii) Uniaxial compressive strength (UCS) of gneiss assessed as 200 $N/mm^2$.

Ultimate allowable bond stress in rock taken as 0.1 UCS up to a maximum of 4.2 $N/mm^2$.

Select safety factor = 3.0

Then working bond strength = $1.4N/mm^2$

and minimum length of fixed anchor to transfer stress from rock to grout is

$$\frac{3610 \times 10^3}{\pi \times 215 \times 1.4} = 3818\text{mm}$$

A fixed anchor length of 6000mm is recommended to allow for variations in rock strength.

(c) <u>Grout to corrugated sheath interface</u>

Select diameter of corrugated sheath as 150mm (this allows a grout annulus of greater than 25mm thickness)

At the working load of the anchor the bond stress at the interface between the grout and the sheath is

$$\frac{3610 \times 10^3}{\pi \times 150 \times 6000} = 1.28\ N/mm^2$$

c.f. The recommended maximum value in CP110 of $2.6N/mm^2$ for ultimate anchorage bond stress for deformed reinforcement in Grade 40 concrete.

R

3. Overall anchor length

Assume resistance to anchor system is provided by a slab of rock with dimensions in plan equal to those for the base of the structure i.e. 77 x 47m.

Total resistance to be provided is 678700kN.

Contribution from "rock strength" across bottom surface of slab taken as $24kN/m^3$ (Hilf,1973)*

i.e. 77 x 47 x 24 = 86900kN

The weight of the rock mass must provide the balance of the resistance.

Submerged weight of rock taken as $16kN/m^3$.

Therefore depth of slab required =

$$\frac{678700 - 86900}{77 \times 47 \times 16} = 10.2m$$

Assume that the slab of rock extends half way down the fixed length.

Then overall length of vertical anchors is

$$10 + \frac{6}{2} = 13m$$

The above calculations for overall anchor length will generally be conservative because no account has been taken of the shear strength around the nominal perimeter of the rock slab which would normally be expected to be present in a massive rock such as Lewisian Gneiss.

It is recommended that the overall stability of the system be assessed using test anchors (see Appendix III).

*HILF, J.W. (1973) Reply to Aberdeen Questionnaire (1972) unpublished.

## APPENDIX III

### Anchor Test Programme

It is strongly recommended that a comprehensive anchor test programme is carried out at an early stage in the project. The location for the tests should be chosen as an onshore site ideally with the Lewisian Gneiss exposed at the surface. The thickness of the rock should be at least 25m and the "structure" of the rock ideally should be similar to the production site or sites.

The principal objectives of the tests should be :-

i) To determine the optimum safe fixed anchorage length for the anchors.

ii) To monitor long term service performance, in order to determine the correct initial lock off loads.

iii) To check practical aspects of anchor construction/installation including handling arrangements, homing, damage during homing, grouting and proof loading.

iv) To obtain information on drilling production and anchor construction rates to form the basis for detailed programming.

It is proposed, to achieve these objectives, that at least 12 No. full scale anchors are installed, proof tested and locked off. The intial phase of the programme would be to install say, 8 No. full scale anchors having varying fixed anchor lengths from 2.0 to 6.0 m to enable calculation of the ultimate rock/ grout bond strength. A representative 4 No. anchors designed using this information would incorporate permanent load cells immediately prior to locking off. The load in these anchors would then be monitored at agreed intervals over a minimum of 5 years.

Throughout the programme details of all operations together with the performance of materials and equipment would be recorded for later analysis so that the preliminary proposals can be updated or modified if necessary.

Our Budget Cost for carrying out such a Test Programme during a period not exceeding 8 weeks is £50,000 at September 1979 prices.

APPENDIX IV

List of References


1. "Ground Anchors in Civil Engineering". G. S. Littlejohn. Ground Engineering, May 1968.

2. "Rock Anchors - State of the Art". G.S. Littlejohn, D.A. Bruce. Foundation Publications Ltd., Brentwood, Essex. 1977.

3. "Soil and Rock Anchors - Examples from Practice". Losinger/VSL publicity.

4. "Ground Anchors at Devonport Nuclear Complex". G.S. Littlejohn and C. Truman Davies. Ground Engineering November, 1974 pp 19-24

5. "Long term performance of high capacity anchors at Devonport". G.S. Littlejohn and D.A. Bruce. Ground Engineering November, 1979.

6. "Soil Anchors". G.S. Littlejohn. I.C.E. Ground Engineering Conference, June 1969 pp.33-44.

7. "Construction, carrying behaviour and creep characteristics of ground anchors". H. Ostermayer. I.C.E. Diaphragm Walls and Anchorages.

8. "Ground Anchors - State of the Art". G.S. Littlejohn. Concrete Society of South Africa, Ground Anchor Symposium,Johannesburg, November,1979

9. "Grouting the Ninian Central Platform's Underbase". Ground Engineering. November, 1978 pp 17-22.

10. "Caisson Underslab Grout" - Dubai Dry Dock. Colcrete Limited, Internal Technical Note.

11. "Acceptable water flows for rock anchor grouting". G.S. Littlejohn. Ground Engineering March 1975 pp 46 - 48.

12. "Anchor Field Tests in Carboniferous Strata". G.S. Littlejohn, D.A. Bruce, W. Deppner. Revue Francaise de Geotechnique. 1978 No.3 pp 82-86.

CTD/SD